ASSESSMENT OF RESISTANCE FACTORS OF A DRILLED SHAFT EMBEDDED IN WEATHERED ROCK FOR LOAD AND RESISTANCE FACTOR DESIGN

Hong-Joon Yoon
Assistant Manager
Incheon Bridge Construction Office,
Korea Expressway Corporation
yoonhj@ex.co.kr

Myoung-Mo Kim, Ph.D.
Professor
Department of Civil and Environmental Engineering,
Seoul National University
geotech@snu.ac.kr

Abstract: *The Load and Resistance Factor Design (LRFD) method is being used increasingly in geotechnical design practice worldwide, and is expected to completely replace the current Allowable Stress Design (ASD) method in the near future. LRFD has advantages over ASD in that it allows the design of superstructures and substructures at a consistently reliable level by quantifying the failure probability based on a reliability analysis. At present, the resistance factors of drilled shafts embedded in rocks suggested in AASHTO only for intact rock conditions. In Korea, where piles are commonly embedded in heavily weathered rocks, the AASHTO-suggested resistance factors of drilled shafts cannot be applied. Thus, this study was conducted to determine the resistance factors of heavily weathered rocks. A reliability analysis was carried out using the FOSM method to evaluate the resistance factors of drilled shafts embedded in weathered rocks in Korea. Both the shaft resistance and the end-bearing resistance from each of the 21 drilled shafts that were installed at four different construction sites were measured from pile load tests. A total of 18 shaft resistance data and 13 end-bearing resistance data were collected under failure conditions of the shaft and the end-bearing for use in the reliability analysis. The computed shaft resistance and end-bearing resistance factors ranged from 0.1 to 0.6 and from 0.2 to 0.5, respectively, depending on the method used to determine the pile resistance. This study was performed as a preliminary research to assess the resistance factors in the design of the pile foundation of the Incheon Bridge.*

Keywords: Load and Resistance Factor Design, Reliability analysis, Drilled shaft

1. INTRODUCTION

Over the past two decades, there has been a general shift from using the Allowable Stress Design (ASD) to using the Limit State Design (LSD) in structural and geotechnical design practice around the world. Among the various types of LSDs, the Load and Resistance Factor Design (LRFD) method is being increasingly used in geotechnical design practice worldwide, and is expected to completely replace the current ASD method in the near future. LRFD has an advantage over ASD in that it allows superstructures and substructures to be designed at a consistently reliable level by quantifying the failure probability based on a reliability analysis. At present, the resistance factors of drilled shafts embedded in rocks suggested in AASHTO [1] only for intact rock conditions.
In this study, therefore, the appropriate resistance factors of drilled shafts embedded in weathered rock were evaluated using pile load test data for 21 drilled shafts at four selected sites and using the properties of weathered rock. The load factors, the bias factors of load and resistance, and the target reliability index were needed to evaluate the resistance factors. The load factors and the bias factors of the load were employed as used in the NCHRP REPORT 507 [10]. It was difficult, however, to accurately evaluate the bias factors of the resistance because there were no reference values to compare them with and there was a greater degree of inherent uncertainty in the geotechnical design than in the superstructure design. In this research, to evaluate the bias factors of the resistance values of a side and an end-bearing, the measured resistance from a pile load test were divided into the side resistance and the end-bearing

resistance according to a load transfer analysis based on the strain gauges and telltale rod measurements. The predicted pile capacities were calculated using various static analysis methods. The target reliability index was estimated based on the reliability index of the current ASD via statistical analysis as well as through a literature review. Through the reliability analysis, the resistance factors were presented for each static analysis method. Because this study was performed as a preliminary research to assess the resistance factors of the pile foundation of the Incheon Bridge, the results of this study were not directly reflected in the design of the pile foundation of the Incheon Bridge.

2. DETERMINATION OF STATISTICAL PARAMETERS

2.1 Measured resistance based on the pile load test

In this study, the resistance factor of the drilled shaft embedded in weathered rock was calculated. Data from conventional axial load tests on drilled shafts were collected. The data summary on the 21 test piles is given in Table 1. The static axial load tests were conducted on the piles that were embedded in moderately to completely weathered rock masses at the selected sites. The diameters of the test piles ranged from 0.4 m to 1.5 m and were socketed to depths that ranged from 0.2 m to 9.0 m.

Table 1: Summary of the drilled shafts data

Site	Number of Test Piles	Rock Description	Socket Dimensions (mm)
K-site	7	Highly weathered, Granite-Gneiss	Diameter: 400 Embedded depth: 200-2,000
S-site	7	Extremely weathered, Granite-Gneiss	Diameter: 400 Embedded depth: 3,000-9,000
N-site	2	Extremely weathered, Volcanic Breccia	Diameter: 1,500 Embedded depth: 5,000
D-site	5	Highly weathered, Gneiss	Diameter: 1,000 Embedded depth: 2,500

It was found from the inspection of the load-settlement curves, however, that few loading tests were conducted wherein the pile base materials failed because of their large axial bearing resistance values. In such cases, an empirical relation was used to estimate the ultimate end-bearing resistance values. The empirical relation between the secant modulus of the q-w curve (k_b) and the ultimate end-bearing resistance (q_{max}) that was applied to this project is shown in Figure 1.

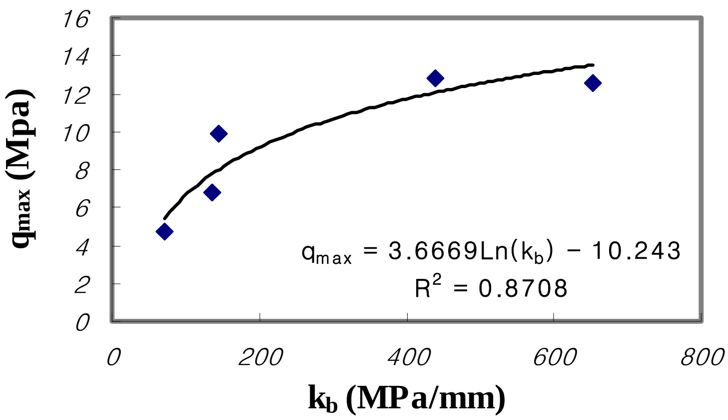


Figure 1: Relationship between k_b and q_{max}

For the load transfer analysis of the piles, strain gauges and telltale rods were installed on the test piles that were located at the D-site and the N-site. The piles that were located at the S-site and the K-site, however, were not equipped with strain gauges, and thus, their load transfers were not measured. The measured resistance values based on the pile load test data are summarized in Table 2.

Table 2: Measured resistance estimated from the pile load tests

Site	Pile No.	Side Resistance (kPa)	End-bearing Resistance (MPa)
K-site	1 (MX)	362	-
	2 (MX)	542	11.0*
	4 (MX)	692	12.8
	5 (MX)	1095	10.4*
	6 (MX)	1312	12.9*
	7 (MX)	1465	12.6
S-site	3 (CX)	428	6.8
	4 (CX)	155.3	7.2*
	5 (CX)	187.2	4.7
N-site	P1 (upper) (CX)	254	-
	P1 (lower) (MX)	330.1	9.9
	P2 (MX)	354.8	10.7*
D-site	2 (lower) (HX)	670	13.3*
	2 (upper) (CX)	510	-
	3 (lower) (HX)	711.1	-
	4 (upper) (HX)	407	-
	4 (lower) (MX)	1136.7	15.0*
	5 (lower) (MX)	1399.3	15.8*

Note: *: End-bearing resistance values, which were indirectly measured from the relationship between the secant modulus (k_b) and the ultimate end-bearing resistance (q_{max}) (Figure 1).

2.2 Predicted resistance based on the static analysis method

The rock mass properties at each site were obtained from field tests (e.g., NX boring, sampling and observation of rock cores, and a pressuremeter test) and a laboratory test (e.g., a uniaxial compressive test and a point load test). Static analyses of the ultimate resistance values of the 21 drilled shafts at the four load test sites were carried out using various design methods (Horvath and Kenny [5], LCPC SETRA [6], Rowe and Armitage [11], Carter and Kulhawy [3], Zhang and Einstein [14], and FHWA IGM [9]). Table 3 presents a summary of the methods that were used for the static analyses of the drilled shafts and the required parameters. The ultimate resistance value that was computed in this method is called the predicted resistance. The result of the calculation of the predicted resistance in this study was omitted in this paper due to space limitations.

Table 3: Static analysis methods used to evaluate the predicted side resistance and end-bearing resistance

Resistance Component	Design Method	Side Resistance and End-bearing Resistance	Parameters Required	Constraints
Side	Horvath and Kenny	$f_{max} = 0.65 p_a (\frac{q_u}{p_a})^{0.5}$	Uniaxial compressive strength of the intact core	Smooth wall
	Rowe and Armitage	$f_{max} = 0.45(q_u)^{0.5}$	Uniaxial compressive strength of the intact core	Smooth wall
		$f_{max} = 0.60(q_u)^{0.5}$	Uniaxial compressive strength of the intact core	Rough wall
	FHWA IGM	$f_{max} = \sigma'_{vi} K_{oi} \tan \phi'_i$	Unit weight, ground water level, SPT N value	Non-cohesive IGM
		$f_{max} = q_u / 2$	Uniaxial compressive strength of the intact core	Cohesive IGM
End-bearing	Zhang and Einstein	$q_{max} = 4.83(q_u)^{0.51}$	Uniaxial compressive strength of the intact core	
	Carter and Kulhawy	$q_{max} = [s^{0.5} + (ms^{0.5} + s)^{0.5}]q_u$	Uniaxial compressive strength of the intact core ,Rock mass properties	Lower bound limit
	LCPC SETRA	$q_{max} = k(p_l - p_o) + \sigma_o$	Pressuremeter test results	k = 1.8 for the rock k = 1.1 for the soil

2.3 Bias factors of resistance

The resistance statistical parameters are represented in this paper in terms of the bias factor of the resistance. The bias factor of the resistance (λ_R) is defined as the ratio of the measured resistance to the predicted resistance. The bias factor was computed using the procedure shown in Figure 2. The result of the calculation of the bias factor in this study was omitted in this paper due to space limitations.

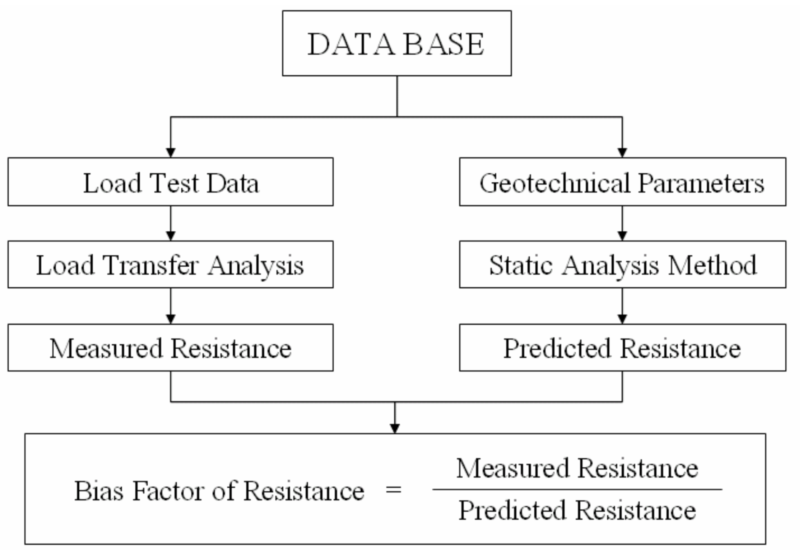


Figure 2: Process of estimation of the bias factor

Table 4 shows the number of test piles, the mean, the standard deviation, and the coefficient of variation (COV_R) of the bias factor, along with the static analysis methods. As shown in Table 4, the mean λ_R, which is the mean value of the bias factors, was used to evaluate the overestimation or underestimation of the actual resistance. Therefore, the static analysis method, in which λ_R equals 1.0, provided the most accurate estimation of the actual resistance. Also, when λ_R was less than 1.0, the static analysis results underestimated the actual resistance. As shown in Table 4, the static analysis methods of Hovath and Kenny [5], the FHWA non-cohesive IGM [9], and Carter and Kulhawy [3] all underestimated the actual resistance values, but those of Rowe and Armitage [11], the FHWA cohesive IGM [9], Zhang and Einstein [14], and LCPC SETRA [6] all overestimated the actual resistance values.
The coefficient of variation (COV_R), which is the ratio of the standard deviation to the mean λ_R, is related to the precision of each static analysis method and was used to evaluate the level of dispersion of the distribution of λ_R. Therefore, a lower COV_R indicates a more efficient design.

Table 4. Mean and standard deviation, and COV_R of the bias factor.

Resistance Component	Design Method	No. of Cases	Mean λ_R	Standard Deviation	COV_R	Remark
Side	Horvath and Kenny	18	1.44	0.70	0.49	
	Rowe and Armitage	18	0.46	0.22	0.48	Smooth wall
		18	0.34	0.17	0.48	Rough wall
	FHWA IGM	9	0.37	0.17	0.45	Cohesive
		18	2.63	1.73	0.66	Non-cohesive
End-bearing	Zhang and Einstein	10	0.63	0.19	0.31	
	Carter and Kulhawy	10	3.50	3.02	0.86	
	LCPC SETRA	5	0.42	0.08	0.20	Rock
		5	0.68	0.14	0.20	Soil

3. TARGET RELIABILITY INDEX

3.1 Reliability level of the current design practice

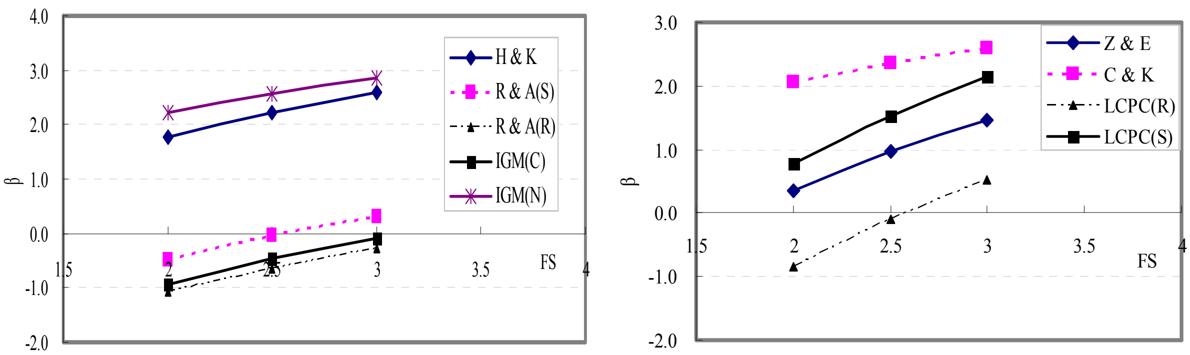
The reliability level of the current design practice was evaluated using the overall factor of safety (FS) that is used in ASD. The reliability index (β) is a function of FS, the dead-to-live-load ratio (Q_D / Q_L), the load statistical parameters, and the resistance statistical parameters. The load statistical parameters were assumed from those used in the NCHRP REPORT 507 [10], and the resistance statistical parameters that were determined in the actual distribution of the bias factor that was obtained from this study, were used. The reliability analysis in this study was performed for the FS values of 2.0, 2.5, and 3.0, because the 2-3 range of the FS is commonly used in practice. Q_D / Q_L depended on the construction material and the span length of the bridge. β, however, did not vary significantly for different load ratios. Therefore, the Q_D / Q_L of 2 according to Nowak [8] was used to calculate the resistance factor because of the relatively small influence of Q_D / Q_L on β.
The β in the current ASD practice was evaluated using the First-Order Second Moment method (FOSM) suggested by Cornell [4]. Table 5 summarizes the results of the reliability analyses of the drilled shafts that were embedded in weathered rock. As shown in Table 5, the calculated β varied widely according to the static analysis method. This implies that different static analysis methods provide different levels of reliability, even though they have the same FS.

Table 5: Reliability indices for the drilled shafts

Resistance Component	Design Method	FS = 2.0	FS = 2.5	FS = 3.0	Remark
Side	H&K	1.78	2.22	2.58	
	R&A	-0.48	-0.04	0.32	Smooth wall
		-1.08	-0.64	-0.28	Rough wall
	IGM	-0.93	-0.47	-0.09	Cohesive
		2.22	2.57	2.86	Non-cohesive
End-bearing	Z&E	0.36	0.96	1.46	
	C&K	2.07	2.36	2.59	
	LCPC	-0.84	-0.09	0.53	Rock
		0.78	1.53	2.15	Soil

Notes: H & K: Horvath and Kenny [5]
R & A (S): Rowe and Armitage [11], smooth wall
R & A (R): Rowe and Armitage [11], rough wall
IGM (N): FHWA IGM [9], cohesionless IGM
IGM (C): FHWA IGM [9], cohesion IGM
Z & A: Zhang and Einstein [14]
C & K: Carter and Kulhawy [3]
LCPC (S): LCPC SETRA [6], soil
LCPC (R): LCPC SETRA [6], rock

Figure 3 shows the variations of the calculated β from the FOSM according to the conventional factor of safety (2, 2.5, and 3.0). Rowe and Armitage [(S) and (R)] [11] and FHWA IGM (C) [9] showed low reliability levels for the side resistance, and LCPC (R) [6] showed a low reliability level for the end-bearing resistance.



(a) Side resistance (b) End-bearing resistance

Figure 3: β versus the factor of safety

3.2 Determination of the target reliability index (β_T)

Meyerhof [7] suggested that the probability of failure (p_f) of driven piles should be between 1×10^{-3} and 1×10^{-4}, which correspond to the β_T between 3 and 3.6. Wu et al. [13] suggested that the β_T for pile systems would be approximately 4, which corresponds to $p_f = 5\times10^{-5}$. Tang et al. [12] reported that the β_T for offshore piles ranges from 1.4 to 3.0, which corresponds to a p_f of between 1×10^{-1} and 1×10^{-3}.

Barker et al. [2] suggested the β_T according to the pile type. Recently, the NCHRP Report 507 [10] suggested that the β_T was 2.33 for redundant piles, which are defined as five or more piles per pile cap, and 3.0 for non-redundant piles, which are defined as four or fewer piles per pile cap.
Based on the reliability analysis results and the abovementioned review of previous researches on β_T, it seems reasonable to establish β_T as between 2.0 and 2.5 for the pile group and as high as 3.0 for a single pile. Usually, the driven piles are constructed as large groups. On the other hand, the drilled shafts are constructed as single or small groups. In this study, therefore, the range of the target reliability index was determined to be 2.5 and 3.0.

4. DETERMINATION OF THE RESISTANCE FACTORS

4.1 Resistance Factors for the drilled shafts

The resistance factor calculation was carried out using FOSM with selected β_T values of 2.5 and 3.0. The calculated resistance factors are shown in Table 6. The calculated resistance factors shown in the table range from 0.1 to 0.6, but the resistance factors that Rowe and Armitage [11] and the FHWA cohesive IGM [9] obtained were relatively small values of around 0.1, because, as shown in Table 4, these static analysis methods overestimate the actual resistance of the ground.

Table 6: Resistance factors for the drilled shafts embedded in the weathered rock

Resistance Component	**Design Method**	β_T **= 2.5**	β_T **= 3.0**	**Remark**
Side	Horvath and Kenny	0.49	0.38	
	Rowe and Armitage	0.15	0.12	Smooth wall
		0.12	0.09	Rough wall
	FHWA IGM	0.14	0.11	Cohesive
		0.60	0.43	Non-cohesive
End-bearing	Zhang and Einstein	0.31	0.26	
	Carter and Kulhawy	0.51	0.34	
	LCPC SETRA	0.26	0.23	Rock
		0.43	0.37	Soil

5. CONCLUSIONS

In this study, a reliability analysis was carried out to evaluate the resistance factors of drilled shafts embedded in weathered rocks in Korea. This study was also performed as a preliminary research to assess the resistance factors in the design of the pile foundation of the Incheon Bridge. The following conclusions are drawn.

1. The reliability index for the safety factors of 2.0, 2.5, and 3.0 of the ASD were estimated using the second moment reliability method, FOSM, and the range of the reliability indices was found to have been -1.08 to 2.86. Based on these results and the literature review that was also performed, the most appropriate target reliability indices for the evaluation of the resistance factors were 2.5 and 3.0.

2. The resistance factors of the drilled shafts that were embedded in weathered rock were evaluated from pile load test data on 21 drilled shafts at four selected construction sites. The resulting resistance factors ranged from 0.1 to 0.6.

3. In this reliability analysis, few test piles were used. Since the reliability of the resistance factors can be enhanced with more quality data, it is essential to continue investigating and updating the resistance factors.

REFERENCES

[1] AASHTO. 2004. LRFD. *Bridge Design Specifications. 3rd edition*. Washington, D.C.
[2] Barker R., Duncan J., Rojiani K., Ooi P., Tan C., and Kim S. 1991. NCHRP Project 343: *Manuals for the Design of Bridge Foundations*. TRB, National Research Council, Washington, D.C.
[3] Carter J.P. and Kulhawy F.H. 1988. *Analysis and design of drilled shaft foundations socketed into rock*. Report EL-5918, Electric Power Research Institute.
[4] Cornell C.A. 1969. *Safety Specifications Based on Second Moment Reliability Analysis*. Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Mass.
[5] Horvath R.G. and Kenny T.C. 1979. Shaft resistance of rock-socketed drilled piers. *Proceedings of the Symposium on Deep Foundations*, ASCE National Convention, Atlanta, pp. 183-214.
[6] LCPC-SETRA. 1985. *Regles de justificaton des fondations sur pieux a partir des resultats des essais pressiometriques*, Ministre de L'Urbanisme du logement et des Transports, Paris (in French).
[7] Meyerhof G. 1970. Safety factors in soil mechanics. *Canadian Geotechnical Journal*, Vol. 7, No. 4, pp. 349-555.
[8] Nowak A. 1999. NCHRP Report 368: *Calibration of the LRFD Bride Design Code*. TRB, Washington, D.C.
[9] O'Neill M.M. and Reese L.C. 1999. *Drilled Shafts: Construction Procedures and Design Methods*. Publication No. FHWA-IF-99-025, U.S. Department of Transportation, Federal Highway Administration.
[10] Paikowsky S., Birgisson B., McVay M., Nguyen T., Kuo C., Baecher G., Ayyub B., Stenersen K., O'Malley K., Chernauskas L., and O'Neill M. 2004. NCHRP REPORT 507: *Load and Resistance Factor Design (LRFD) for Deep Foundations*. TRB, Washington, D.C.
[11] Rowe R.K. and Armitage H.H. 1987. A design method for drilled piers in soft rock. *Canadian Geotechnical Journal*, Vol. 24, pp. 126-142.
[12] Tang W., Woodford D., and Pelletier J. 1990. Performance reliability of offshore piles. *Proceedings of the 22nd Annual Offshore Technology Conference*. Houston, TX, Paper No. OTC 6379, Offshore Technology Conference, Richardson, TX, Vol. 3, pp. 299-308.
[13] Wu T., Tang W., Sangrey D., and Baecher G. 1989. Reliability of offshore foundations: State of the art. *Journal of Geotechnical Engineering*, Vol. 115, No. 2, pp. 157-178.
[14] Zhang L. and Einstein H.H. 1988. End-bearing resistance of drilled shafts in rock. *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 124, No. 7, pp. 574-584.